

FINAL REPORT

**COMPOSITE ACTION IN A STEEL GIRDER SPAN WITH PRECAST DECK PANELS:
THE I-81 BRIDGE OVER THE NEW RIVER IN RADFORD, VIRGINIA**

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ABSTRACT

Two parallel bridges carry I-81 north and south over the New River in southwest Virginia, near the city of Radford. The bridges are identical in design and have been in place since 1985. In recent years, a number of maintenance issues have been reported, primarily related to cracking of the cast-in-place topping over partial-depth precast deck panels. A study was undertaken to determine the influence of the observed deterioration on the structural capacity of the affected bridge spans.

The analysis indicated that the full potential of the composite slab-girder system is no longer being realized. Continued deterioration of the deck is likely, especially given the frequency of heavy truck traffic on this structure and the inherent vibration. It appears that the presence of precast cast-in-place deck sections has reduced the overall stiffness of the deck as compared to the original design. The movement, in conjunction with a poor deck panel support detail, is likely to cause a continual maintenance problem, as additional precast panels begin to move and fracture of the cast-in-place topping occurs.

As a potential mitigation option, replacement of the fiber bolster material between the top flange of the girders and the precast panels with more rigid steel shims and/or concrete is recommended to increase the bearing surface of the panels, reduce vertical displacement of panel edges, and minimize dynamic impact at the joints.

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INTRODUCTION

Two parallel bridges carry I-81 north and south over the New River in southwest Virginia, near the city of Radford. The bridges are identical in design and have been in place since 1985. In recent years, a number of maintenance issues have been reported, primarily related to cracking of the cast-in-place topping over partial-depth precast deck panels.

Design of Structure

Each of the parallel sister structures comprises 10 spans totaling 1,657.5 ft and 1,599.5 ft in length for the southbound lanes (SBLs) and northbound lanes (NBLs), respectively. Each structure carries two lanes of rural interstate traffic, with a large volume of heavy multi-axle trucks and tractor-trailers. Each structural span is composed of five haunched steel plate girders at 9-ft spacing, supporting a total deck width of 41 ft 11 in. The original design included continuous cast-in-place concrete decks. The order of placement was required to be upgrade, and epoxy sealing of construction joints was specified. During construction, substitution of partial-depth prestressed, precast deck panels with a cast-in-place wearing surface was permitted (see Figure 1). The panels were 8 ft wide by 8 ft long. The as-built prestressed slab thickness was documented as $3\frac{3}{4}$ in, with cast-in-place (CIP) concrete topping bringing the total slab thickness to approximately $8\frac{1}{2}$ in.



Figure 1. Underside of Precast Deck and Girder Supports System

History of Maintenance Problems

An interview with the bridge engineer for the Virginia Department of Transportation's (VDOT) Salem District revealed that maintenance has become a continuous and worsening problem with regard to this structure. Specifically, particular spans have developed significant spalling of the CIP concrete layer in the deck, and in some cases, deterioration under heavy interstate traffic has progressed such that prestressing strands in some precast deck panels have been exposed. The resident and district personnel have repeatedly performed remedial patching and repairs to maintain the deck. With the progressive deterioration came concerns about the influence on the structural capacity of the affected spans. Specifically, the question was raised concerning the degree of composite action between the precast/CIP deck sections and the supporting girders.

Previous Field Evaluation

In response to the inquiry about composite action and structural capacity, the structures research team at the Virginia Transportation Research Council (VTRC) conducted a field test. The test involved monitoring strain in the steel girders at midspan while loading the span with a tandem dump truck of known weight and axle dimensions. The load tests were conducted at highway speed to capture the effects of dynamic loading. Strain data were captured using real-time monitoring equipment and stored for further analysis. A summary report was generated,

which included a discussion of the observed strains, estimated load distribution factors for the structure, and comparisons to a finite-element model of the structure (Roche et al., 2001).

PURPOSE AND SCOPE

The purpose of this study was to determine the influence of the observed deterioration on the structural capacity of the affected bridge spans.

METHODS

Document Review

To enable an understanding of the maintenance issues encountered with this structure, available documentation was reviewed. Available data included the original construction plans (not indicating the precast deck panels), periodic bridge inspection reports and documentation, and correspondence between VDOT and Federal Highway Administration (FHWA) personnel regarding the maintenance history of the structure, as provided by the VDOT Salem District's Structure and Bridge Office.

Prior VDOT correspondence specifically highlighted the detail of the precast deck panel and girder interface as contributing to the observed maintenance issues (Napier, 2000). A bridge survey report contained a field sketch of the as-built dimensions of the interface (see Figure 2) (Barnhart, 1999).

Figure 2 shows that the precast panels are bearing on a narrow strip of fiber bolster material along the outer edge of each top girder flange. Shear studs extend from the top face of the girder within the channel formed by the adjacent panels, and this space contains concrete, cast monolithically with the concrete topping layer above the panels. According to the field sketch, it appears that prestressing strands from the panels were not truncated at the face of the panel but were allowed to extend a short distance and were subsequently encapsulated in the CIP concrete.

A memorandum (Napier, 2000) indicates that where the edge of the panels bear on the fiberboard and CIP concrete, the panels do not extend beyond the fiberboard sufficient distance to provide permanent support. Napier states that the 1982 specifications (American Association of State Highway and Transportation Officials [AASHTO], 1977) under which the structure was built, and the as-built details of the structure, do not meet current recommended industry standards in this regard. Napier cites a 1987 memorandum from FHWA, which emphasizes, "the most significant detail for deck panels is to insure proper positive bearing of the deck panels on the beams" (Gordon, 1987). Napier's recommendations were to continue monitoring the structure, conduct dynamic load testing of the structure, and consider the viability of replacing the fiberboard with a more positive support material (Napier, 2000).

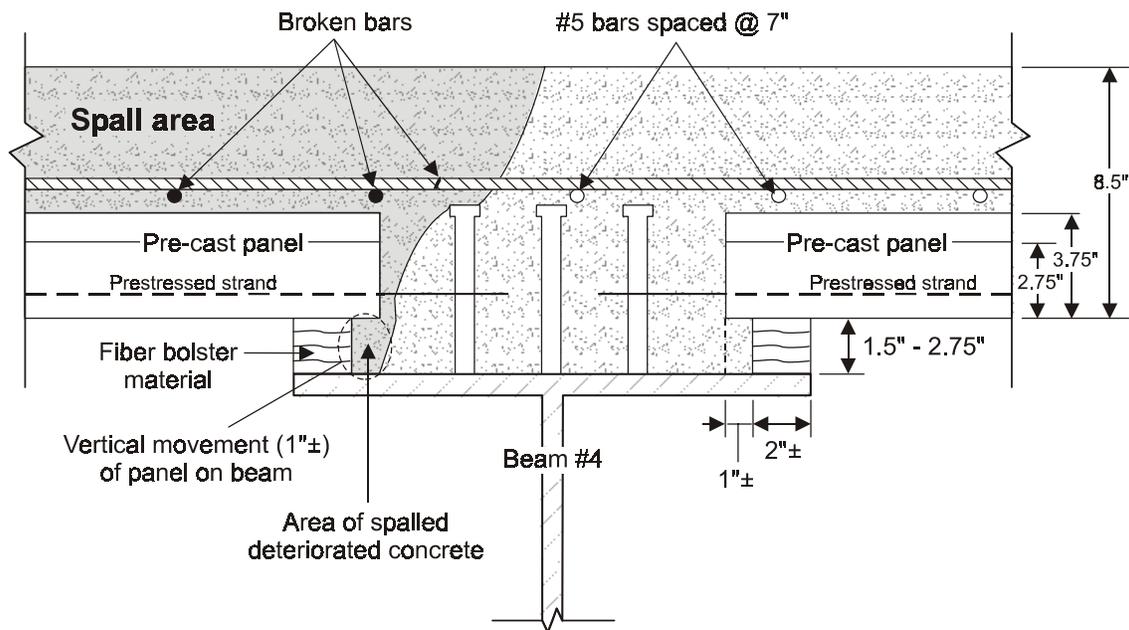


Figure 2. Schematic of As-Built Deck-to-Girder Interface

Field Testing

In August 2002, a research team from VTRC, with the assistance of VDOT's Salem and Bristol district personnel and personnel from Virginia Tech, instrumented Span 8 of the structure carrying NBLs for a series of load tests. Although Span 3 of the structure has exhibited the most severe and consistent deterioration to date, it is relatively inaccessible for the purpose of load testing since it is located directly above the river. Span 8 has displayed similar deterioration but has not progressed as rapidly. Also, like Span 3, it is located as the third span in from the bridge abutment. Incidentally, similar deck cracking has been documented in the sister structure carrying SBLs of I-81, though deterioration has not yet progressed to the extent observed on this structure.

As in the previous field test conducted in August 2000, weigh-in-motion (WIM) strain gages were used to measure mid-span strain in the longitudinal direction in the top and bottom flanges of each of the five girders of Span 8 NBL (see Figure 3). The length of this span was 170 ft. In addition, vertical displacement gages were used to determine simultaneous real-time deflection of each girder under load. The data were logged for later analysis and comparison to previous field tests. The data sample rate was 1,200 readings per second for all gages.

A series of experimental loadings, or "passes" were conducted, concentrating the load alternately in the left "passing" lane, the right "travel" lane, and the right shoulder. In 2002, the load vehicle was a tandem, three-axle dump truck loaded near legal capacity, with an approximate gross weight of 49,000 lb. The same vehicle had been employed in August 2000, but the total load was 47,500 lb at that time. For reference, the girders were numbered 1 through 5, with girder 1 representing the left-most girder when viewed in the direction of traffic.



Figure 3. Strain and Displacement Gages at Midspan of Span 8 NBL

RESULTS

Observed Girder Strains

A total of 12 passes were made, and data were recorded. Of these, four passes were identified, two in the right (travel) lane and two in the left (passing) lane, during which a “clean” pass was achieved in that observations and the strain data indicated clearly the passing of the single truck load, unhindered by the influence of other large vehicles on the subject or adjacent spans of the structure. As an example, Figure 4 presents a plot of the strain response in the top and bottom flanges of Girder 1 during a left lane pass. It is interesting to note the presence of a 3 to 5 Hz vibration, which appears superimposed on the fundamental deflection and strain induced by the passing truck. This vibration reflects the complex dynamic nature of truck loading on continuous multi-span structures of this length. The vibration, in addition to other “noise” induced by traffic on non-adjacent spans of the bridge, makes it difficult to relate the strain data directly to the load induced by the test vehicle. However, more direct comparison of strain data could still be done to assess composite action within the slab-girder section.

The peak tensile strains in the bottom flange of each girder were derived from the strain data obtained during a given pass. For the four passes evaluated, Table 1 presents the peak tensile strain in each girder and some simple statistics about the data. The greatest strain noted was approximately 71 microstrain ($\mu\text{m}/\text{m}$). Assuming a modulus of elasticity for the steel in the girder of 29×10^6 psi, the corresponding maximum observed stress would be approximately 2.06 ksi, which is considered a very low stress for this material and presents no danger of yielding.

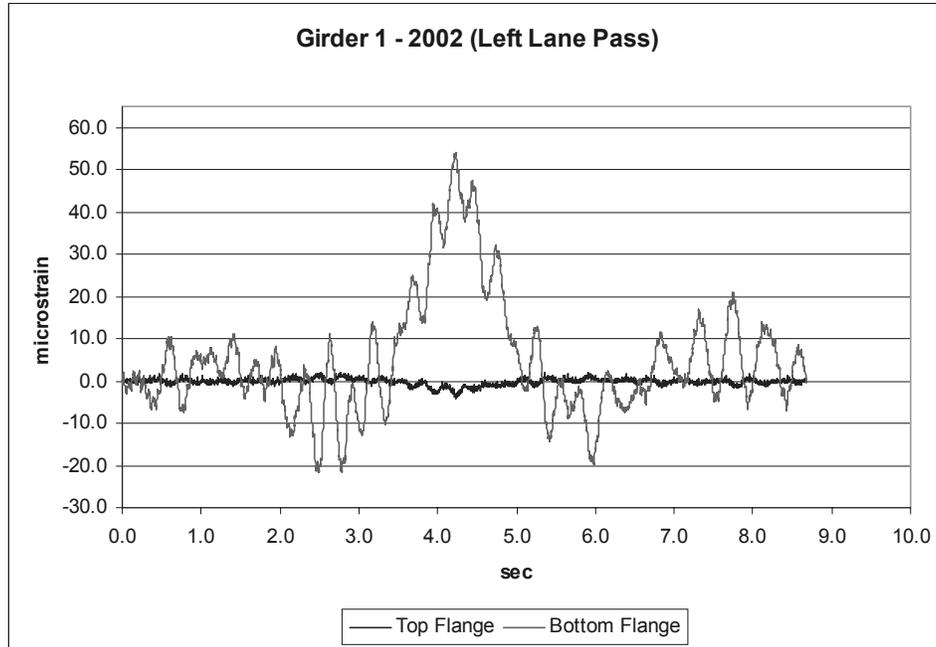


Figure 4. Typical Strain Response from Load Test

Table 1. Observed Peak Tensile Strains ($\mu\text{m}/\text{m}$)

Test		Girder				
		1	2	3	4	5
3	Right lane, 63 mph	23.1	34.2	53.0	40.9	33.6
4	Right lane, 66 mph	24.1	37.7	49.5	46.0	42.8
5	Left lane, 65 mph	53.6	70.7	43.6	28.5	26.7
6	Left lane, 63 mph	46.1	61.3	33.3	21.6	27.4

Using the peak measured strain at the bottom (tension) flange for a given pass, and the time-correspondent strain at the top (compression) flange at the same point in time, the depth of the neutral axis (NA) of the slab-girder system under load was estimated. A summary of NA depth for each girder, as measured from the top surface of the deck, is indicated in Table 2. The NA depth ranged from 13 to 21 in, with the shallowest NA depth, indicating the greatest degree of composite action between the slab and girder, observed in Girder 1. The deepest NA location was observed in Girder 5, indicating less contribution from the concrete/precast slab in moment resistance in this location.

Table 2. Depths to Neutral Axes (in) from Field Strain Data

Test	Girder					
	1	2	3	4 ^a	5	
August 2000						
7	13.3	19.1	22.1	-	12.3	
8	10.8	18.6	25.8	-	12.1	
9	14.9	20.3	16.8	-	12.0	
10	4.5	19.4	19.3	-	11.7	
Average	10.9	19.3	21.0	-	12.0	
Standard Deviation	4.57	0.74	3.86	-	0.24	
Coefficient of Variance	42.0%	3.8%	18.4%	-	2.0%	
August 2002						
3	Right Lane, 63 mph	13.0	20.3	14.0	17.5	19.7
4	Right Lane, 66 mph	13.6	20.1	14.2	16.7	21.1
5	Left Lane, 65 mph	14.6	16.3	16.1	16.6	18.5
6	Left Lane, 63 mph	13.6	14.1	21.6	15.3	19.4
Average		13.7	17.7	16.5	16.5	19.7
Standard Deviation		0.65	3.04	3.56	0.93	1.08
Coefficient of Variance		4.7%	17.2%	21.6%	5.6%	5.5%

^aBottom flange strain gage inoperative during the August 2000 load test.

Observed Girder Deflections

In order to determine the load distribution in Span 8, deflections were measured simultaneously with the previously mentioned girder strains. The deflections were measured using deflectometers attached to the bottom flange of each girder in Span 8 at mid-span. The deflectometers were 0.125-in-thick aluminum plate cantilevered from the bottom flange of the steel girders and were connected to a dead weight on the ground by a thin steel wire. The aluminum plate was instrumented with a full bridge of strain gages, and tip deflection of the plate was calibrated to resistance across the full bridge to within 0.002 in. Girder deflection measurements were continuously recorded with the previously mentioned data acquisition system.

Because of the heavy truck traffic on the bridge (which essentially meant the bridge vibrated continuously during the measurements), the accuracy of the discussed measurements was less than desired. The vibration of the bridge resulted in “at rest” strain measurements of as much as ± 25 microstrain and deflection measurements of ± 0.1 in. This effect was included in the analysis of the data, which resulted in the elimination of some truck crossings and data from inclusion. Figure 5 is a typical plot of girder deflections, as measured during a left-lane pass.

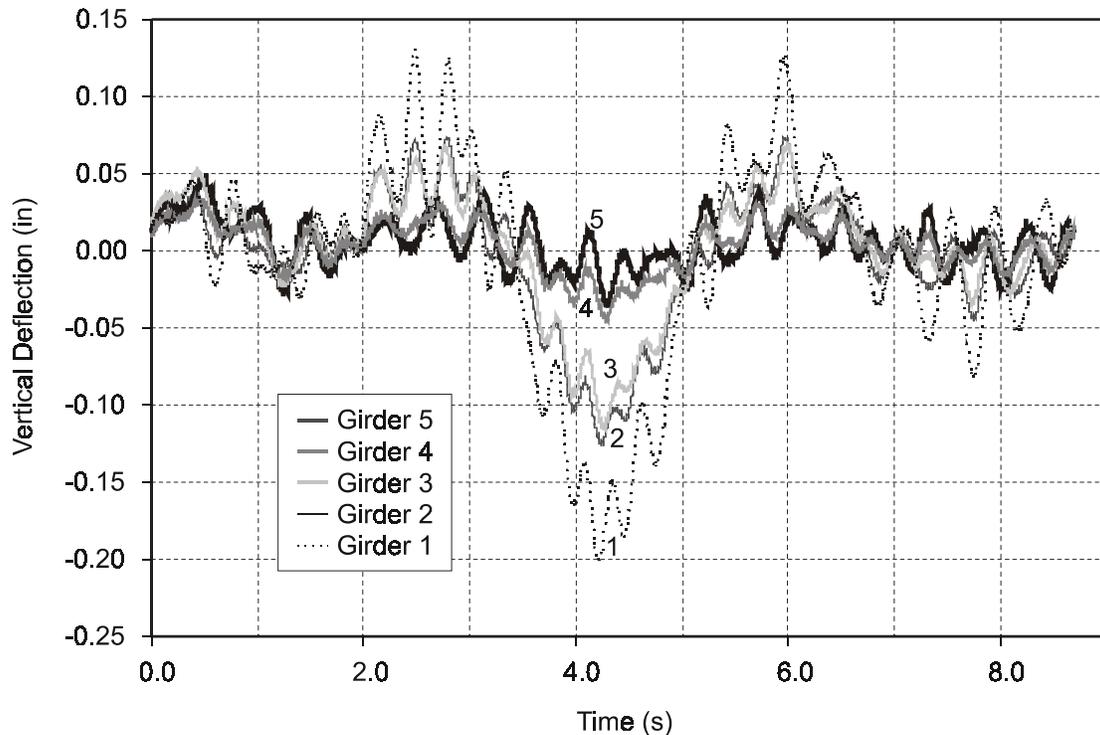


Figure 5. Typical Plot of Girder Deflection During Left Lane Pass

DISCUSSION

Comparison of Girder Strains

Prior load tests were conducted on the Span 8 NBL in August 2000 (Roche, 2001). The previous tests were conducted using the same system and gage layout as described in this study. Thus, direct comparison of top and bottom flange strains in the girders was possible.

Figure 6 and Figure 7 present comparative data from two passes, each conducted in August 2000 and again in August 2002 for left- and right-lane loading, respectively. The plots present the peak strains in the bottom flange and the corresponding simultaneous strain in the top flange for each girder in a given pass. The bottom flange strain gage on Girder 4 was inoperative during the August 2000 load testing.

As shown in Figure 6, representing truck passes in the left lane, the strain in the bottom flange of Girder 1 was significantly higher during both passes in 2002 as compared to the two passes in 2000. A slight increase in bottom flange strain was also observed in Girder 2 over time. Strains in the other girders appeared essentially similar over time.

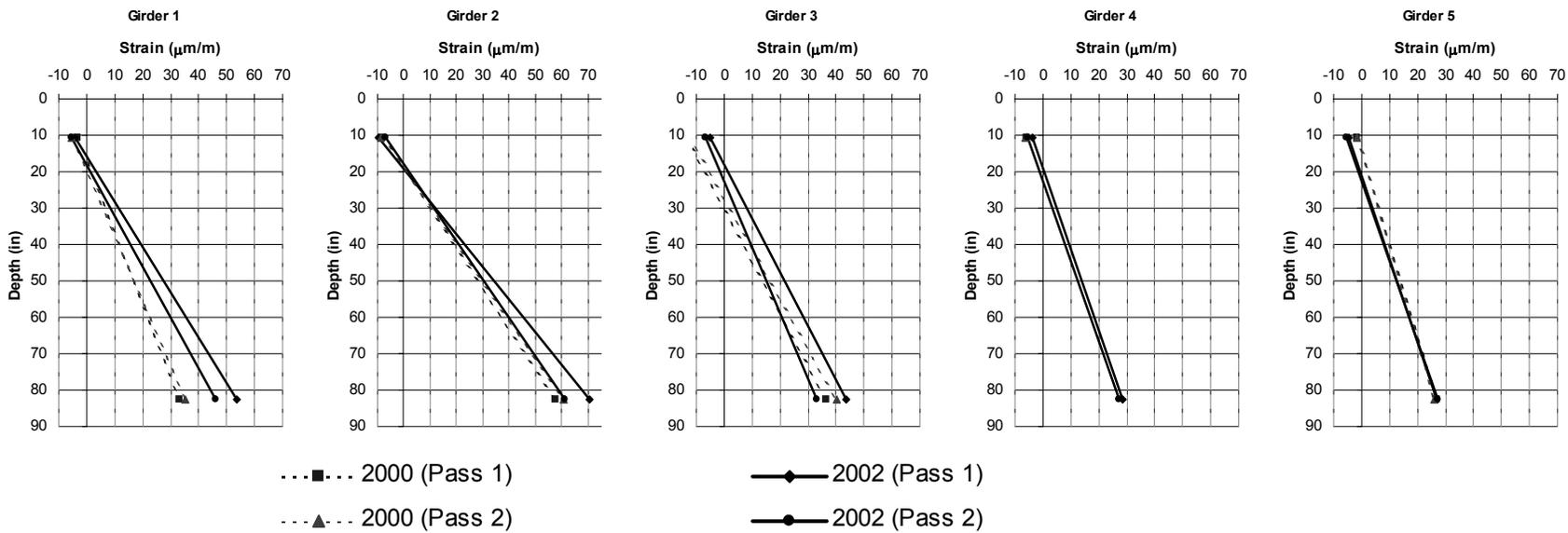
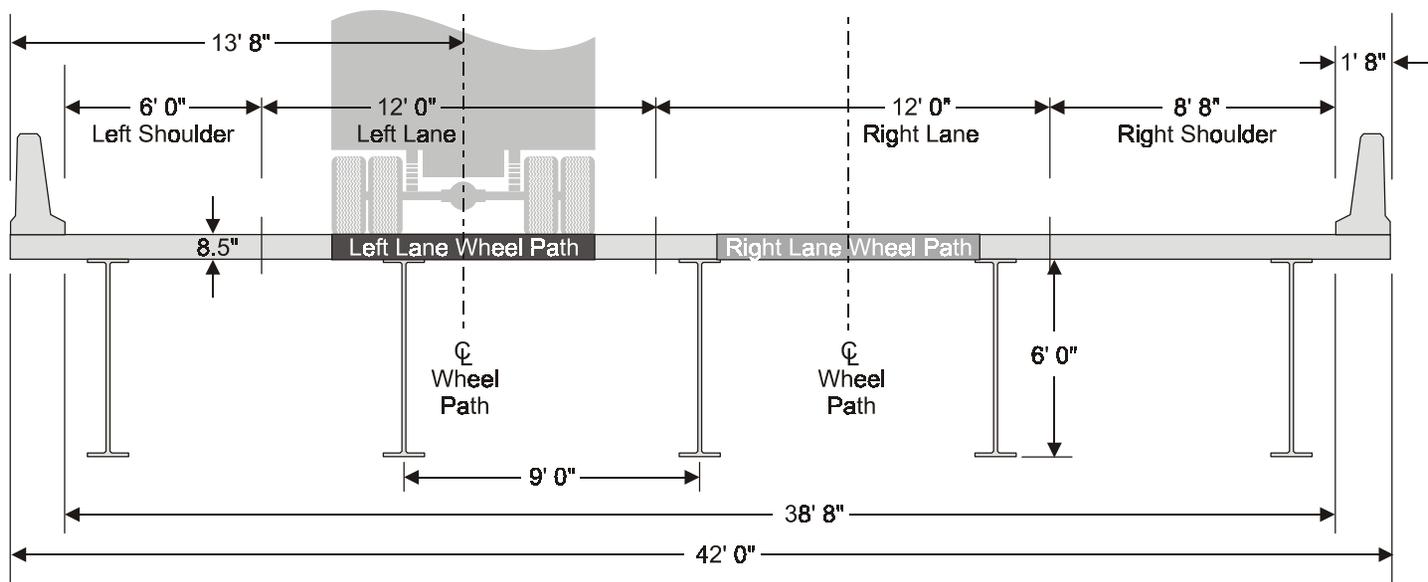


Figure 6. Comparison Strain Distributions by Girder Over Time: Left-Lane Passes

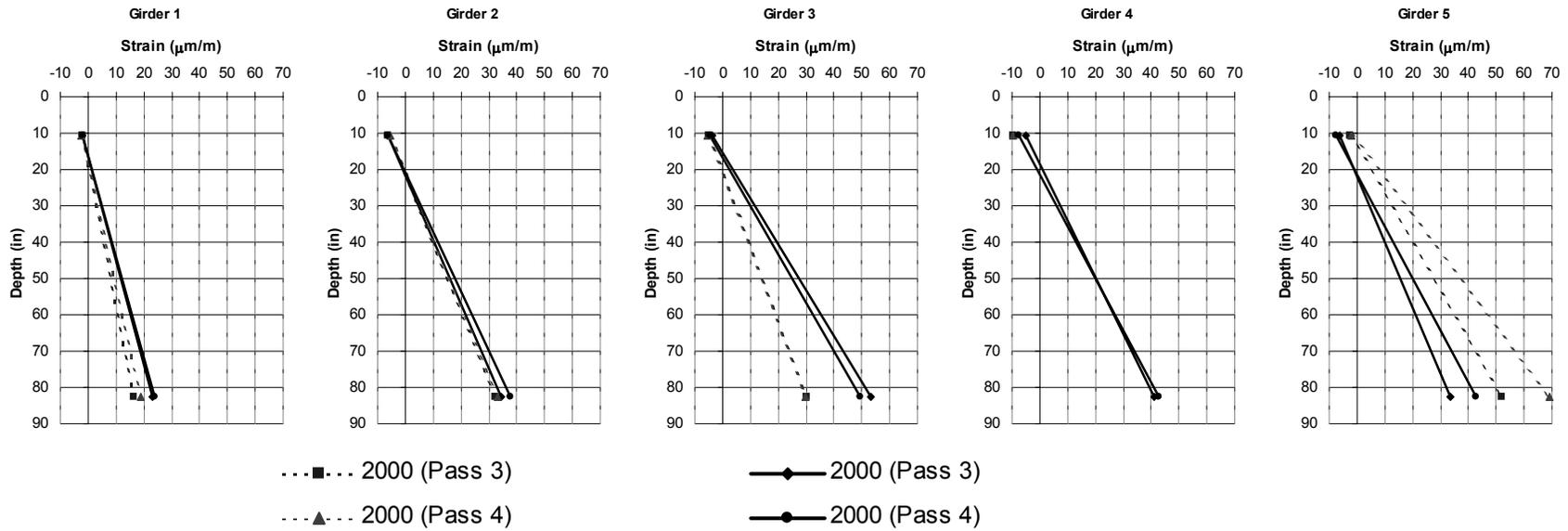
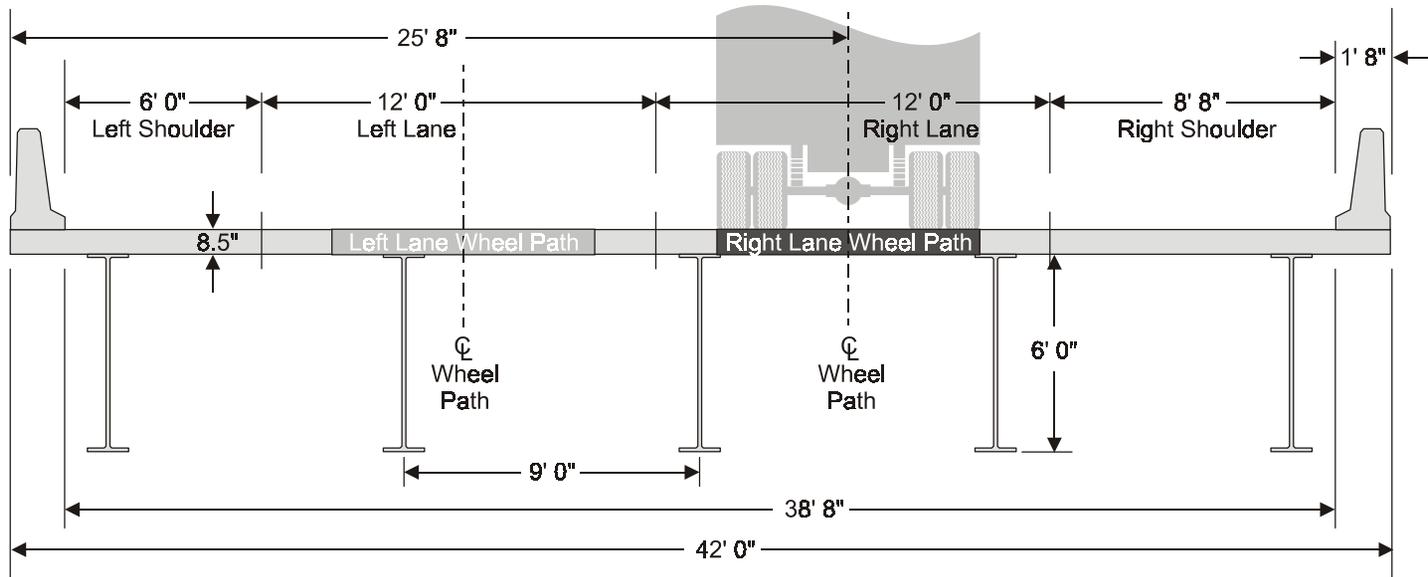


Figure 7. Comparison Strain Distributions by Girder Over Time: Right-Lane Passes

Figure 7, which represents right-lane truck passes, can be compared in the same manner. Girder 3 exhibited significantly greater strain in the lower flange in 2002 as compared to 2000. This might be interpreted as indicating a change in section properties for this slab-girder section. However, a reverse of this trend was observed in Girder 5, where the overall strain in the bottom flange was less in 2002 than in 2000. The wheel path outlined in Figure 7 is somewhat idealized, and the actual path of the truck will vary from one pass to the next. Thus, the increased strain in one girder and simultaneous decrease in another may be related more to the location of the passing truck relative to each of the girders at the two times of testing than to a change in section behavior. Therefore, simple comparisons of strain data are inadequate to assess the situation fully.

Load Distribution

In addition to the degree of individual slab-girder interaction, the effective load distribution through the slab to the various girders was assessed by determining an estimate of the load distribution factor. The girder distribution factor (GDF) for a steel plate girder bridge (as per AASHTO Standard Specifications, 1996) is $S/5.5$ where S is the girder spacing in feet. Thus, for a 9-ft girder spacing, GDF would be 1.64. However, the AASHTO GDFs ($S/5.5$) are for use with a wheel line (which is half of a truck). Hence, the GDFs for a whole truck are about half of what is obtained using $S/5.5$, and the fraction of a whole truckload (both wheel lines) for which each girder would be designed is 0.82. This value includes a factor to account for the presence of multiple trucks (pairs) crossing a bridge.

GDFs were calculated from the test data using both strains from the WIM gages and the deflectometers. The GDF is defined as the fraction of the load in the heaviest loaded girder and is determined by dividing the strain or deflection of the heaviest loaded girder from a truck crossing by the sum of strains or deflection in all girders during that crossing. The GDFs for single truck crossings in the left and right lanes ranged from 0.25 to 0.48. To simulate the presence of side-by-side trucks crossing the bridge, the results from the single truck crossings were combined. These combined results resulted in a maximum GDF of 0.81. The measured GDFs for single truck crossings were 50%, or less than the design value. The combined or multiple-truck crossing GDF was approximately equal to the design value.

Comparison to Calculated Composite Behavior

To compare the observed strain data to an anticipated strain within the slab-girder section, several scenarios were considered, reflecting degrees of composite action, as shown in Figure 8. The first scenario (a) assumes complete interaction among the built-up steel girder, the precast panels on both sides, and the CIP concrete deck surface. Subsequent scenarios consider progressive loss of section performance through loss of bond with one or both precast panels (b and c, respectively), followed by fracture and loss of action of one or both side of the CIP concrete (d and e, respectively), and finally complete loss of load transfer to the deck (f).

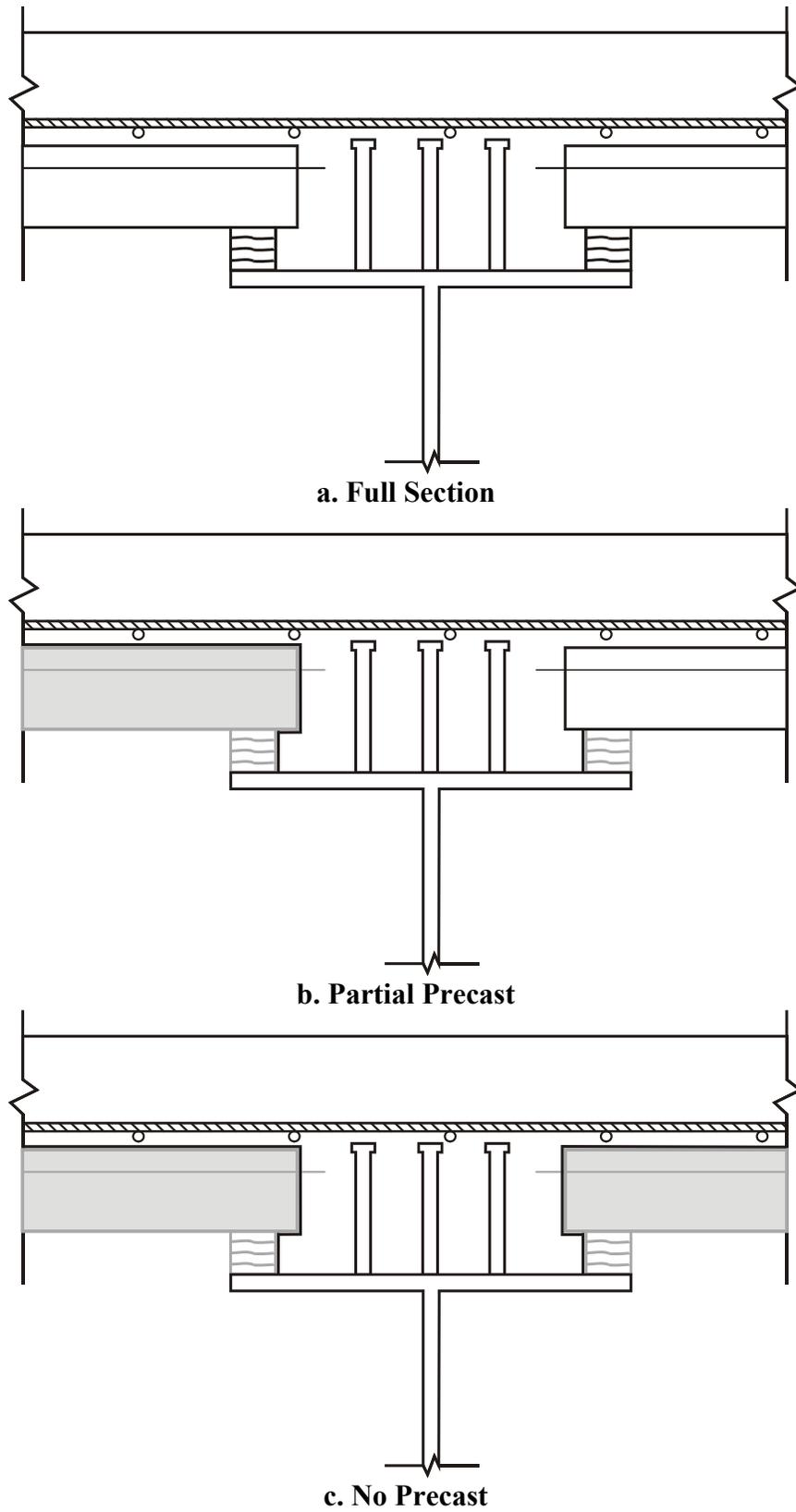


Figure 8. Degree of Composite Behavior between Girder and Precast/Cast-in-Place (CIP) Deck (*continues*)

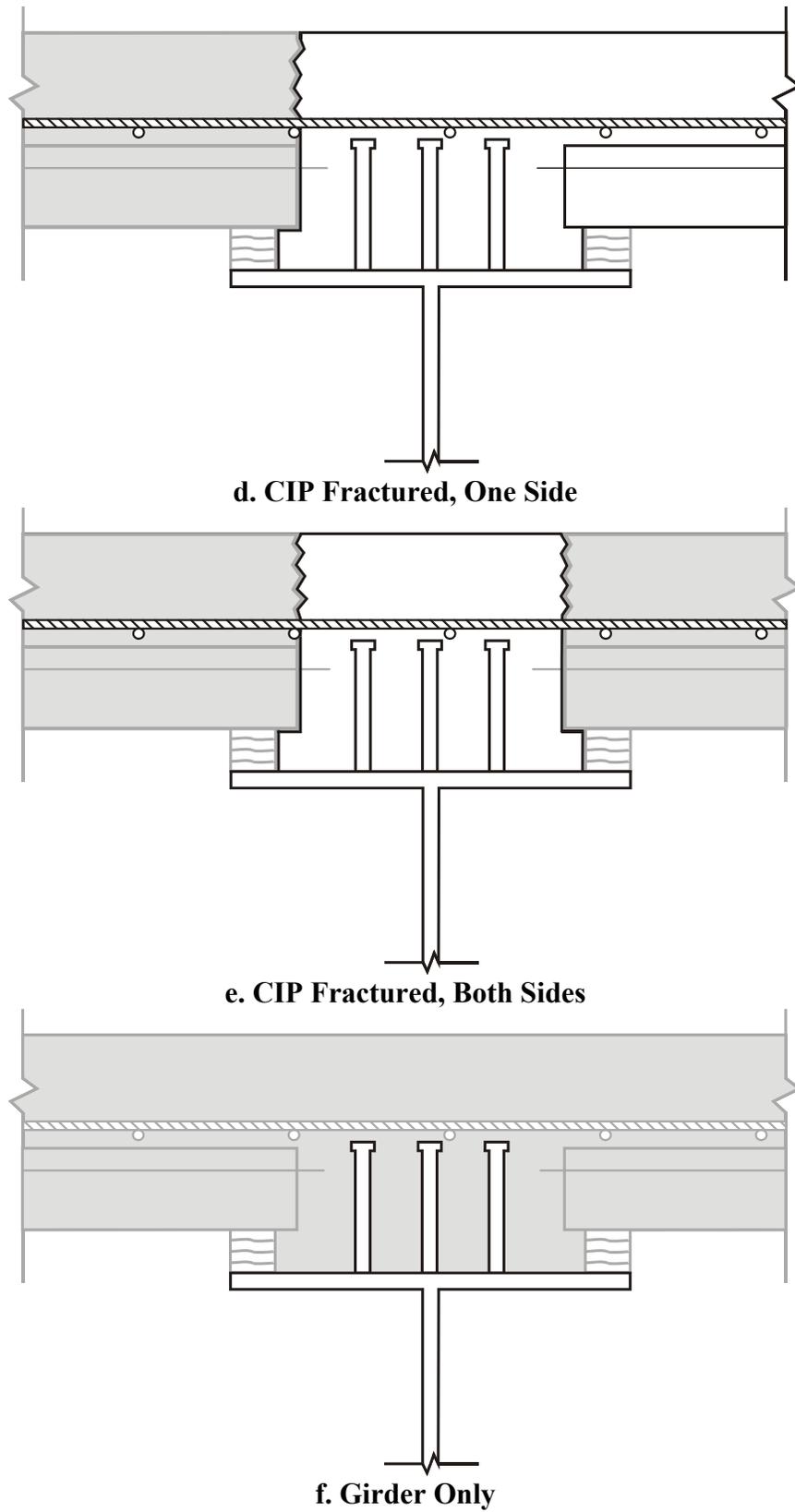


Figure 8 (cont'd) Degree of Composite Behavior between Girder and Precast/CIP Deck

The depth to NA was predicted based on the effective slab-girder cross section in each scenario. The progression of scenarios indicates that as composite action between the deck and the girder is reduced, the depth of the NA will increase. For the analysis, the concrete strength was assumed to be 4,500 psi on average, which translates to a modular ratio (E_s/E_c) of 7.6. Therefore, under ideal conditions, assuming an effective slab width of 108 in (equal to the girder spacing) and full composite behavior, the NA depth for an interior girder is calculated to be 9.1 in. Girders 1 and 5 are exterior girders, and the section properties have been calculated accordingly, assuming an effective slab width of 89.5 in and neglecting parapets. The resulting NA depth for exterior girders was predicted to be 11.1 in. In contrast, if the slab were assumed to contribute nothing to the capacity of the system, the NA for both interior and exterior girders would be expected to shift to 40.8 in depth.

Table 3 compares the NA depth for each girder, as determined in the tests, to the calculated NA depth for the various scenarios outlined previously. The calculations indicate that the average NA location is deeper than would be expected for a fully bonded composite system. In all cases, the average calculated NA depth was beneath the elevation of the slab, extending into the top of the girder. As shown in bold text, the NA depth is in the range more representative of partial or full loss of bond with the precast deck panels, or possibly some fracture of the CIP concrete topping. Although these results do not definitively confirm the loss of composite action between the deck and girders, they do suggest that the contribution of the deck to overall structural capacity is less than anticipated. However, for each girder, sufficient continuity remains between the components such that the girder is not carrying the full moment of the section.

As mentioned previously, this structure had a nearly consistent low-frequency vibration as a result of its continuous steel construction. The structure was originally designed with a full-depth CIP deck. The partial-depth precast panels were apparently introduced as a change during construction to minimize either time or cost. Although the testing and analysis herein does not specifically address the vibration issue, it seems logical that the substitution of the precast panels, which inherently introduced significant vertical and horizontal cold joints within the slab, have decreased the stiffness of the deck as compared to the original design and have exacerbated the vibration inherent in a long-span continuous steel structure. The indications suggesting loss of bond of the precast panels are consistent with this argument.

Table 3. Comparison of Neutral Axes Depths (in)

Scenario	Girder				
	1	2	3	4 ^a	5
Measured average (2000)	10.9	19.3	21.0	-	12.0
Measured average (2002)	13.7	17.7	16.5	16.5	19.7
Full section	11.1	9.1	9.1	9.1	11.1
Partial precast	13.2	11.1	11.1	11.1	13.2
No precast	16.0	13.9	13.9	13.9	16.0
CIP fractured, one side	22.2	20.5	20.5	20.5	22.2
CIP fractured, both sides	32.6	32.6	32.6	32.6	32.6
Girder only	40.8	40.8	40.8	40.8	40.8

^aBottom flange strain gage inoperative during the August 2000 load test.

Computer Modeling of I-81 Bridge

To provide additional insight into the response of this structure, a simplified finite element model of the bridge was developed and subjected to various loadings; the response was then evaluated. The commercial finite element code ANSYS was employed in this phase of the study to predict dynamic response. The finite element model developed was a modification of an earlier model that was used to study the effect of various design parameters on dynamic response. In the current study, the model necessarily included several simplifications and approximations as a result of the complexity of the actual bridge in terms of size and construction details. Accordingly, the results presented should be taken as only relative order of magnitude response values and not actual numerical displacement values.

To develop a simplified finite element model of the bridge that would be operational within the existing ANSYS code, the following simplifications were made. First, the span lengths of the various spans were rounded to the nearest 10 ft and all of the elements were selected to have a length of 10 ft. This permitted easy generation of the full 10 spans. Next, the transverse section was represented by a single girder line that consisted of a single interior girder and a corresponding segment of slab. This approximation, in effect, reduced the bridge to a 10-span beam model whose cross section approximated a section of the actual structure. The deck was assumed uniform and composite with the girder. Boundary conditions were assumed to be pinned at one abutment and on rollers at all other support points.

The applied load consisted of a single 50,000-lb load moving at a constant 65 mph velocity across the bridge. This transient analysis was accomplished by incrementing the location of the load consistent with the assigned velocity. At each load position, the displacement response was calculated for every node point across all of the spans. The maximum values of displacement at the center of each span for a given load path are shown in Table 5.

Table 4. Finite-Element Model of Girder Deflections (Load = 50,000 lb, Speed = 65 mph)

Span	Node No.	Minimum Deflection (in)	Maximum Deflection (in)
1	7	-0.6244	0.3347
2	22	-1.011	0.3927
3	39	-1.042	0.4154
4	56	-0.9964	0.4295
5	73	-1.038	0.4558
6	90	-1.065	0.4169
7	107	-1.045	0.4421
8	124	-1.076	0.4243
9	141	-1.049	0.4662
10	158	-1.284	0.4622

From the calculated response values, it would appear that the maximum displacement is approximately the same order of magnitude in all spans except for Spans 1 and 10. The predicted maximum displacement in Span 1 is on the order of 40% less than the average in most of the spans whereas the predicted maximum displacement in Span 10 is approximately 25% higher. Because of the many approximations made in developing the model and the limitations of the applied load, care should be used in drawing conclusions from these data. Nevertheless, it would appear that Span 10 may be susceptible to larger displacements under a moving load.

Unfortunately, this analysis did not lend any insight into why Spans 3 and 8 have shown the earliest signs of decks deterioration. It appears that varying quality and local effects, along with the poor deck support detail described previously, have made deck deflections and the resulting damage somewhat unpredictable.

CONCLUSIONS

- *The full potential of the composite slab-girder system is no longer being realized.* Deterioration has not yet progressed to the point that the girders are carrying the entire load of the spans. Variations in strain over time do not clearly indicate a decline in moment capacity since previous tests for this span.
- *Continued deterioration of the deck is likely, especially given the frequency of heavy truck traffic on this structure and the inherent vibration.* It appears that the presence of precast cast-in-place deck sections has reduced the overall stiffness of the deck as compared to the original design. The movement, in conjunction with a poor deck panel support detail, is likely to cause a continual maintenance problem, as additional precast panels begin to move and fracture of the cast-in-place topping occurs.

RECOMMENDATIONS

1. *As a means of slowing the rate of deterioration, it might be possible to replace the fiber bolster material between the top flange of the girders and the precast panels with more rigid (and more supportive) steel shims and/or formed concrete, shotcrete, or dry-packed cement mortar.* This would increase the surface area at the perimeter of the panels where load is transferred to the girders. Since the steel/concrete fill would be rigid, whereas the fiber bolster material is compressible, vertical displacement of panel edges under truckloads would be reduced or eliminated and dynamic impact at these joints could be minimized.
2. *This mitigation strategy may not be sufficient to arrest the deterioration, so continued frequent inspections are warranted whether retrofit is attempted or not.* It is likely that the bridge will continue to suffer maintenance problems as a result of the poor construction detail, and replacement of the deck or structure may be necessary.

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